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मार्गदर्शी सिद्धान्त

भाग 4 भूगर्भीय सामर्थ्य सुचकांक

**Quantitative Classification
System of Rock Mass —
Guidelines**

Part 4 Geological Strength Index (GSI)

ICS 93.020

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भारतीय मानक ब्यूरो
BUREAU OF INDIAN STANDARDS

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FOREWORD

This Indian Standard (Part 4) was adopted by the Bureau of Indian Standards, after the draft finalized by the Rock Mechanics Sectional Committee had been approved by the Civil Engineering Division Council.

This standard has been published in four parts. The other parts in the series are:

- | | |
|--------|--|
| Part 1 | Rock mass rating (RMR) for predicting engineering properties |
| Part 2 | Rock mass quality for prediction of support pressure in underground openings |
| Part 3 | Determination of slope mass rating (SMR) |

The geological strength index (GSI) is introduced for both hard and weak rock masses. Experienced field engineers and geologists generally show a liking for a simple, fast, yet reliable classification which is based on visual inspection of geological conditions. Past experiences suggest that a classification system should be non-linear for poor rocks as strength deteriorates rapidly with weathering. Further, increased applications of computer modelling have created an urgent need for a classification system tuned specially to computer simulation of rock structures.

The difference between the peak and residual strength of a rock mass with non-persistent joints is larger than that of a rock mass with persistent joints. The implication is that a drop of GSI from peak to residual values is larger for rock masses with non-persistent joints. Besides rock bridges, rock asperity interlocking also contributes to the difference between peak and residual strengths.

For the purpose of deciding whether a particular requirement of this standard is complied with, the final value, observed or calculated, expressing the results of a test or analysis shall be rounded off in accordance with IS 2 : 1960 'Rules for rounding off numerical values (*revised*)'. The number of significant places retained in the rounded off value should be the same as that of the specified value in this standard.

Indian Standard

QUANTITATIVE CLASSIFICATION SYSTEM OF ROCK MASS – GUIDELINES

PART 4 GEOLOGICAL STRENGTH INDEX (GSI)

1 SCOPE

This standard (Part 4) covers the procedure for obtaining the geological strength index (GSI) and to estimate the rock strength parameters required for engineering analysis of underground structures and other structures on rock.

2 REFERENCES

The standards given below contain provisions, which through reference in this text, constitutes provisions of this standard. At the time of publication the editions indicated were valid. All standards are subject to revision and parties to agreements based on this standard are encouraged to investigate the possibility of applying the most recent editions of the standards given below:

IS No.	Title
4880 (Part 6) : 1971	Code of practice for design of tunnels conveying water: Part 6 Tunnel supports
7317 : 1993	Code of practice for uniaxial jacking test for deformation modulus of rock (<i>first revision</i>)
9143 : 1979	Method of determination of unconfined compressive strength of rock materials
13365 (Part 1) : 1998	Quantitative classification systems of rock mass — Guidelines Rock mass rating (RMR) for predicting engineering properties
(Part 2) : 1992	Rock mass quality for prediction of support pressure in underground openings

3 PROCEDURE

3.1 Geological Strength Index (GSI)

Geological Strength Index (GSI) is correlated with RMR and Q as follows:

$$\text{GSI} = \text{RMR}'_{89} - 5 \text{ for GSI} \geq 18$$

$$\text{or } \text{RMR} \geq 23 \quad \dots (1)$$

$$= 9 \ln Q' + 44 \text{ for GSI} < 18 \quad \dots (2)$$

where

$$Q' = \text{modified rock mass quality} \\ [\text{IS 13365 (Part 2)}] \quad \dots (3) \\ = [\text{RQD}/J_n] \cdot [J_r/J_a], \text{ and}$$

RMR'_{89} = rock mass rating according to [IS 13365 (Part 1)] taking ground water rating as 15 and joint adjustment rating as 0.

NOTE — Sometimes, there is difficulty in obtaining RMR in poor rock masses. The Q' may thus be used more often specially in openings in the weak rocks.

Using chart for GSI (*see* Fig. 1) a rock mass is classified by visual inspection alone. In this classification, there are six main qualitative classes of rock masses [*see* IS 4880 (Part 6)] and are as follows:

- a) Intact or massive;
- b) Blocky;
- c) Very blocky;
- d) Blocky/folded;
- e) Crushed; and
- f) Laminated/sheared.

Further, discontinuities are classified into five surface conditions which are similar to joint conditions in RMR [IS 13365 (Part 1)] and are as follows:

- 1) Very good,
- 2) Good,
- 3) Fair,
- 4) Poor, and
- 5) Very poor.

Now a block in the matrix of 6×5 of Fig. 1 is picked up according to actual and undisturbed rock mass classification and discontinuity surface condition. Then corresponding GSI is read.

GSI chart is quantified by incorporating the rock block volume (V_b) formed by the joints or discontinuities and the joint condition factor (J_c). The suggested quantification is also shown in Fig. 1. The block volume (V_b), affected by the joint set spacing and persistence, can broadly be known by the joint spacing given for six different rock classes in Fig. 1. The value of joint condition factor (J_c), controlled by joint roughness, weathering, and infilling material, can be obtained by the following correlation:

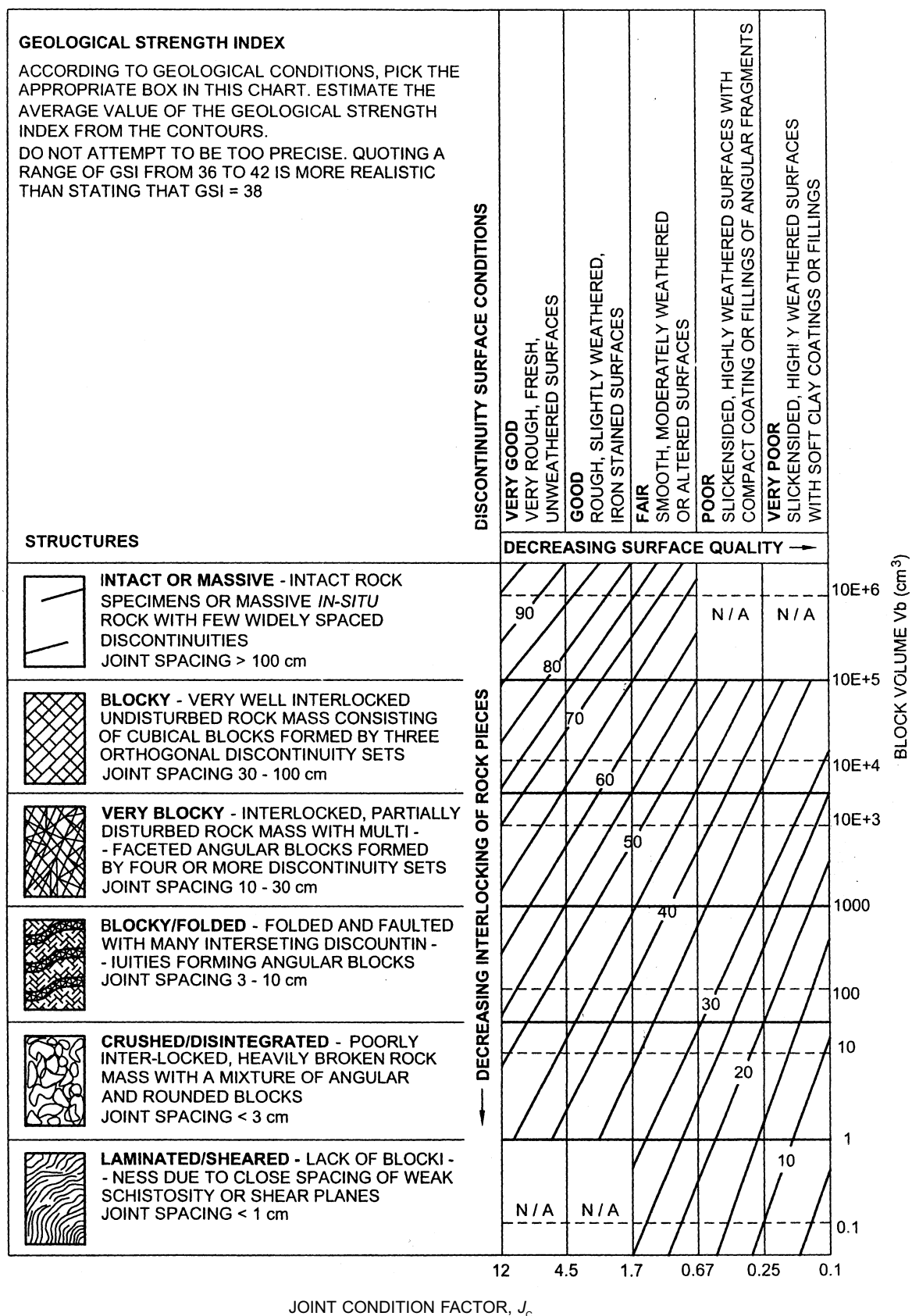


FIG. 1 ESTIMATE OF GEOLOGICAL STRENGTH INDEX GSI BASED ON VISUAL INSPECTION OF GEOLOGICAL CONDITIONS. MODIFICATION IN TERMS OF ITS QUANTIFICATION BY BLOCK VOLUME AND JOINT CONDITION FACTOR IS ALSO SHOWN ON RIGHT SIDE

$$J_C = \frac{\int J_W J_S}{J_A} \quad \dots (4)$$

where

J_W = large-scale joint or discontinuity waviness, in metre, from 1 to 10 m (*see* Table 1);

J_S = small-scale smoothness, in centimetre, from 1 to 20 cm (*see* Table 2); and

J_A = joint alteration factor (*see* Table 3).

NOTES

1 It is recommended to estimate a range of values of GSI (or RMR) in preference to a single value. This Practice has a significant impact on design of slopes and excavations in rocks. Experience shows that drastic degradation in GSI, RMR and Q values is found to occur in openings after squeezing and rock bursts. That is what one see in openings. Hence the need for evaluating the GSI of rock mass in the undisturbed condition ($D = 0$). Back-analysis of both a model (polyaxial strength criterion) and its parameters (from the observed behaviour of rock structures) is an ideal method of the rock mass characterisation.

2 While using Fig. 1 specially for crushed/sheared as well as disintegrated rocks, special caution should be exercised. In case of thickly laminated rocks, consideration should be given to overall hardness of the rock. In this case reference may be made to higher categories in Fig. 1.

Based on the proposed quantitative chart (*see* Fig. 1), and using surface fitting techniques, following equation is used for calculation of GSI from J_C and V_b .

$$GSI(V_b, J_C) = \frac{26.5 + 8.79 \ln J_C + 0.9 \ln V_b}{1 + 0.015 + 1 \ln J_C - 0.0253 \ln V_b} \quad \dots (5)$$

where J_C is a dimensionless factor defined by Equation 4 and block volume V_b , in cm^3 .

3.2 Generalized Strength Criterion

Following non-linear equation is the generalized strength criterion for undisturbed rock masses:

$$\sigma_1 = \sigma_3 + \int q_c \left[m_b \frac{\sigma_3}{q_c} + s \right]^n \quad \dots (6)$$

where

σ_1 = maximum effective principal stress;

σ_3 = minimum effective principal stress;

q_c = UCS of rock material (intact) for standard NX size core or *see* IS 9143

m_b = reduced value of the material constant m_r ,

$$= m_r \cdot \exp \left[\frac{GSI - 100}{28 - 14D} \right]; \text{ and} \quad \dots (7)$$

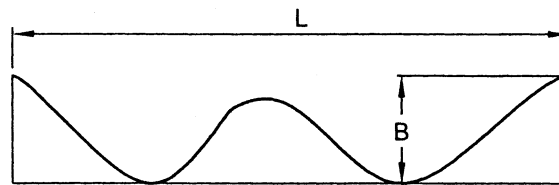
m_r = rock material constant to be found from triaxial tests on rock cores.

The s and n are constants for the rock mass given by the following relationships:

$$s = \exp \left[\frac{GSI - 100}{9 - 3D} \right] \quad \dots (8)$$

$$n = \frac{1}{2} + \frac{1}{6} \left(e^{-GSI/15} - e^{-20/3} \right) \quad \dots (9)$$

Table 1 Terms to Describe Large-Scale Waviness, J_W
(Clause 3.1)



Undulation = $1/L$

L = Length between maximum amplitudes

Sl No.	Waviness Terms	Indulation = $\frac{\alpha \times 100}{L}$ Percent	Rating for Waviness, J_w
i)	Interlocking (Large-scale)	—	3
ii)	Stepped	—	2.5
iii)	Large undulations	>3	2
iv)	Small to moderate undulation	0.2 – 3	1.5
v)	Planar	< 0.3	1

Table 2 Terms to Describe Small-Scale Smoothness, J_s
(Clause 3.1)

SI No.	Smoothness Terms	Description	Rating for Smoothness J_s
(1)	(2)	(3)	(4)
i)	Very rough	Near vertical steps and ridges occur with interlocking effect on the joint surface	3
ii)	Rough	Some ridge and side-angle are evident; asperities are clearly visible; discontinuity surface feels very abrasive (rougher than sandpaper grade 30)	2
iii)	Slightly rough	Asperities on the discontinuity surfaces are distinguishable and can be felt (like sandpaper grade 30 - 300)	1.5
iv)	Smooth	Surface appear smooth and feels so to touch (smoother than sandpaper grade 300)	1
v)	Polished	Visual evidence of polishing exists. This is often seen in coating of chlorite and specially talc	0.75
vi)	Slickensided	Polished and striated surface that results from sliding along a fault surface or other movement surface	0.6 - 1.5

Table 3 Rating for the Joint Alteration Factor, J_A
(Clause 3.1)

SI No.	Description	J_A
(1)	(2)	(3)
i)	1) Clean joints	i) Healed or welded joints: Softening, impermeable filling, for example quartz, epidote, etc
		ii) Fresh joint walls: No coating or filling on joint surface, surface staining only (rust)
	2) Alteration of joint wall	i) The joint surface exhibits one class higher alteration than the rock
		ii) The joint surface shows two classes higher alteration than the rock
	3) Coating or thin filling	i) Coating of friction materials without clay, such as sand, silt, calcite, etc
		ii) Coating of softening and cohesive minerals, such as clay, chlorite, talc, etc
ii)	Filled joints with partly or no contact between the rock wall surfaces	1) Filling of friction material without clay, such as sand, silt, calcite, etc (non-softening)
		2) Hard filling of softening and cohesive materials, such as compacted clay materials
		3) Filling of soft clay materials having medium to low over-consolidation of filling
		4) Swelling clay materials: Filling material exhibits clear swelling properties

NOTE — For avoiding double accounting, ground water condition and *in-situ* stresses are not considered in GSI as these are accounted for in computer models. Further, GSI assumes that the rock mass is isotropic. Therefore, only cores without weak planes should be tested in triaxial cell to determine q_c (UCS) and m_i (see Equation 7) as GSI down-grades strength according to schistosity. This classification reduces many uncertainties in rock mass characterisation. Heavy blasting creates new fractures. Therefore, an undisturbed rock mass should be inspected for classification.

D is a disturbance factor which depends upon the degree of disturbance to which the rock mass has been subjected by blast damage and stress relaxation. It varies from 0.0 for undisturbed *in-situ* rock masses to 1.0 for very disturbed rock masses (see Table 4). While using disturbance factor D , values specified in Table 4 should be used judiciously.

NOTES

1 Actual value of D is a function of rock mass quality as well as blasting quality. The value of D for controlled blasting in hard jointed rocks may be significantly different. Same is true for uncontrolled blasting in above rock types.

2 The correlations for 's' are valid for rock slopes and open pit mines only, and not for structurally controlled rock slopes and

transported rock-fill slopes. For tunnels and caverns, there is an enormous strength enhancement.

3.3 Linear Strength Parameters

Linear strength criterion for a rock mass is expressed as follows:

$$\sigma_1 - \sigma_3 = q_{\text{cmass}} + A \sigma_3 \quad \dots (12)$$

where

q_{cmass} = uniaxial compressive strength of the rock mass = $2 c_p \cos \phi_p / (1 - \sin \phi_p)$;

c = cohesion of the rock mass;

$A = 2 \sin \phi_p / (1 - \sin \phi_p)$; and

ϕ_p = peak angle of internal friction of the rock mass.

Strength parameters c_p and ϕ_p depend upon σ_3 . Average values of c_p and ϕ_p with $D = 0$ are given in Fig. 2 and Fig. 3 respectively for assessment. Table 5 lists typical values of m_r for various types of rock materials.

The angle of dilatancy (Δ) of a rock mass after failure is recommended approximately as

$$\begin{aligned}\phi &= (\phi / 4) && \text{for GSI} = 75 \\ &= (\phi / 8) && \text{for GSI} = 50 \\ &= 0.0 && \text{for GSI} \leq 30\end{aligned} \quad \dots (13)$$

3.4 Modulus of Deformation

The modulus of deformation (E_d), in GPa, of rock mass is found from the following correlation:

$$E_d = E_r \left[0.02 + \frac{1 - D/2}{1 + \exp((60 + 15D - GSI)/11)} \right] \dots (14)$$

where

E_r = modulus of elasticity of intact rock, in GPa.

The elastic modulus of rock mass (E_e), in GPa, is obtained from the unloading cycles of the uniaxial jacking tests (see IS 7317). It is correlated for both dry and saturated rock mass as follows.

$$E_e = 1.5Q^{0.6} E_r^{0.15} \quad \dots (15)$$

where

Q = rock mass quality [see IS 13365 (Part 2)].

NOTES

1 The strength and deformation parameters estimated from the GSI system are very close to those obtained from *in-situ* tests. Back-analysis of observed displacements in openings may give more realistic values of the design parameters including disturbance factor by trial and error procedure.

2 Caverns (width, $B > 15$ m) should be located in nearly dry and non-squeezing ground, with $Q > 1$ and $E_d > 2$ GPa (see Equation 14) generally except in shear zones but $H < 350 Q^{1/3}$ m, where ' H ' is thickness of overburden above the tunnel.

3 The Equation 15 is for the dynamic analysis of concrete dams under seismic conditions and machine (generator, etc) foundations on the rock masses.'

3.5 Parameters for Intact Schistose Rocks

Cohesion along joints is needed for wedge analysis or computer modelling. Cohesion along bedding planes or planar continuous joints (longer than 10 m) may be negligible. However, cohesion along discontinuous joints (assumed continuous in the wedge analysis) may be the same as cohesion (c_p) of the rock mass. The cohesion of rock mass is due to the cohesion of the discontinuous joints. Furthermore, the ratio of c and cohesion of rock material (see Fig. 2) may be of the same order as the area of intact rock bridges per unit area of the discontinuous joints.

NOTE — The geological strength index, GSI and RMR take into account the orientation of joints. To avoid double accounting for joint orientation in both UCS and GSI, upper bound value of q_c and m_r for rock cores with nearly horizontal planes of weakness for estimating m_b , s , and E_d for jointed rock masses are to be used.

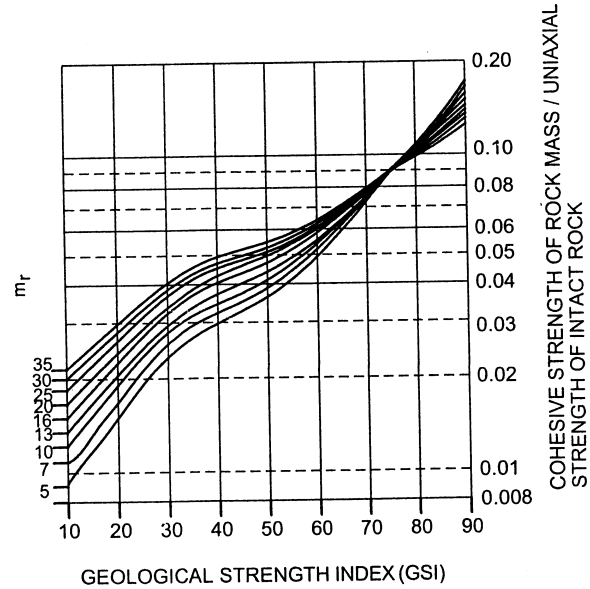


FIG. 2 RELATIONSHIP BETWEEN RATIO OF COHESIVE STRENGTH OF ROCK MASS TO UNIAXIAL COMPRESSIVE STRENGTH ON INTACT ROCK (C/Q_c) AND GSI FOR DIFFERENT m_r VALUES FOR $D = 0.0$

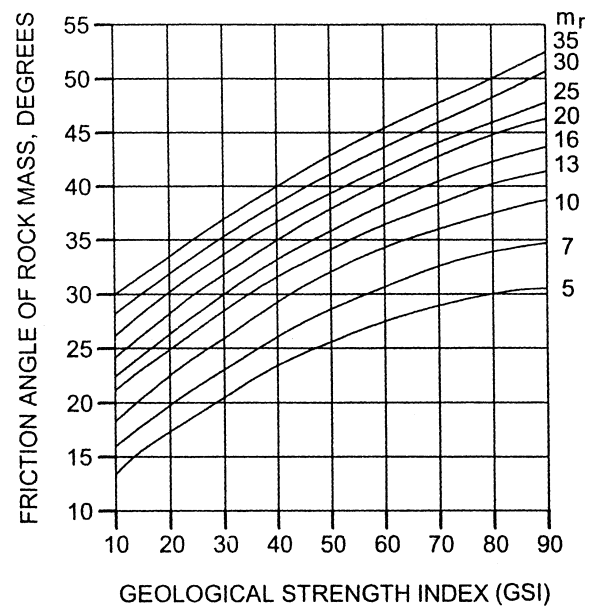
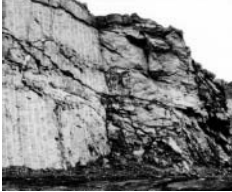


FIG. 3 FRICTION ANGLE ϕ OF ROCK MASS FOR $D = 0.0$ FOR DIFFERENT GSI AND m_r VALUES

Table 4 Guidelines for Estimating Disturbance Factor, D

(Clause 3.2)

Sl No.	Appearance of Rock Mass	Description of Rock Mass	Suggested Value of D
(1)	(2)	(3)	(4)
i)		Excellent quality controlled blasting or excavation by tunnel boring machine results in minimal disturbance to the confined rock mass surrounding a tunnel	$D = 0.0$
ii)		Mechanical or hand excavation in poor quality rock masses (no blasting) results in minimal disturbance to the surrounding rock mass. Where squeezing problems result in significant floor heave, disturbance can be severe unless a temporary invert, as shown in the photograph, is placed	$D = 0.0$ $D = 0.5$ No invert
iii)		Very poor quality blasting in a hard rock tunnel results in severe local damage, extending 2 or 3 m, in the surrounding rock mass	$D = 0.8$
iv)		Small scale blasting in civil engineering slopes results in modest rock mass damage, particularly if controlled blasting is used as shown on the left hand side of the photograph. However, stress relief results in some disturbance	$D = 0.7$ Good blasting $D = 1.0$ Poor blasting
v)		Very large open pit mine slopes suffer significant disturbance due to heavy production blasting and also due to stress relief from overburden removal In some softer rocks, excavation can be carried out by ripping and dozing and the degree of damage to the slopes is less	$D = 1.0$ Production blasting $D = 0.7$ Mechanical excavation

NOTES

1 Experience in the design of slopes in very large open pit mines has shown that criterion for undisturbed in situ rock masses ($D = 0.0$) results in shear strength parameters that are too optimistic. The effects of heavy blast damage as well as stress relief due to removal of the overburden of the rock mass results in disturbance of the rock mass. It is recommended that the 'disturbed' rock mass parameters with $D = 1.0$ in Equation 7 and Equation 8 are more appropriate for slopes in these rock masses.

Thus, uniaxial compressive strength of a rock mass obtained from Equation 6 is,

$$q_{\text{cmass}} = q_c \cdot s^n \quad \dots (10)$$

and uniaxial tensile strength of a good rock mass is

$$q_{\text{umass}} = -\frac{sq_c}{m_b} \quad \dots (11)$$

2 Some extremely weak rocks (for example, sand rock, silt stone, clay stone, unconsolidated rocks in lesser Himalaya) with uniaxial compressive strength less than 1.0 MPa in dry or saturated condition, will behave as soils. These rocks should be classified as soils according to Indian Standards and not GSI.

3 The failure criterion, which assumes isotropic rock mass behaviour, should be applied to those rock masses in which there are sufficient numbers of closely spaced joints with similar properties.

4 GSI is not applicable to the anisotropic rock masses. There wedge failure analysis should be carried out in joint-controlled stability of slopes, caverns and tunnels, using strength parameters along discontinuities. The rock mass rating or GSI along failure plane may be much less than that on the rock slope in distress.

Table 5 Estimated Values of the Constant M_r for Intact Rock Material
(Values are for guidance only)
(Clause 3.3)

SI No.	Broad Rock Class	Rock Types	Values
(1)	(2)	(3)	(4)
i)	Sedimentary	Anhydrite	13
		Breccia	20
		Chalk	7
		Claystone	4
		Coal	8 - 21
		Conglomerate	22
		Greywacke	18
		Gypstone	16
		Limestone	8 - 10
		Sandstone	19
		Siltstone	9
		Amphibolite	25 - 31
		Gneiss	33
		Hornfels	19
		Marble	9
		Migmatite	30
		Mylonites	6
		Phyllites	10
ii)	Metamorphic	Quartzite	24
		Schist	4 - 8
		Slates	9
		Agglomerate	20
		Andesite	19
		Basalt	17
		Breccia(Volcanic)	18
		Dacite	17
		Diorite	2
		Dolerite	1
		Granite	33
		Granodiorite	3
iii)	Igneous	Gabbro	27
		Norite	22
		Obsidian	19
		Rhyolite	16
		Tuff	15

NOTE — These values are for intact rock specimens tested normal to bedding or foliation. The value of m_r shall be significantly different if failure occurs along a weakness plane.

3.6 Estimation of Residual Strength of Rock Masses

To extend the GSI system for estimation of rock mass residual strength, the original GSI value is to be adjusted based on the two major controlling factors in the GSI system, that is, block volume (V_b) and joint condition factor (J_c) to reach the residual values.

3.6.1 Residual Block Volume

If a rock experiences post-peak deformation, the rock in the broken zone is fractured and consequently turned into a poor and eventually ‘very poor’ rock. Hence the properties of a rock mass after extensive straining should be derived from the rock class of ‘very poor rock mass’ in the RMR system or ‘disintegrated’ in the GSI system.

For the residual block volume, it is observed that the post-peak block volume are small because the rock mass has experienced tensile strain and shear fracturing. After the peak load, the rock mass becomes less interlocked, and is heavily broken with a mixture of angular and partly rounded rock pieces.

The failed rock mass blocks are 1 cm to 5 cm in size. The rock mass is disintegrated along a shear zone. As such, following criterion is recommended for estimating the residual block volume V_b^r :

- If $V_b > 10 \text{ cm}^3$, V_b^r : (in disintegrated category) $= 10 \text{ cm}^3$
- If $V_b < 10 \text{ cm}^3$, $V_b^r = V_b$

3.6.2 Residual Joint Condition Factor

The residual joint surface condition factor J_c^r is calculated from

$$J_c^r = \frac{J_w^r J_s^r}{J_A^r} \quad \dots (16)$$

where J_w^r , J_s^r and J_A^r are residual values of large-scale waviness, small-scale smoothness and joint alteration factor respectively. The reduction of J_w^r and J_s^r are based on the concept of mobilized joint roughness and the equations are given as

$$\text{If } \frac{J_w}{2} < 1, J_w^r = 1; \text{ Else } J_w^r = \frac{J_w}{2}, \quad \dots (17)$$

$$\text{If } \frac{J_s}{2} < 0.75, J_s^r = 0.75; \text{ Else } J_s^r = \frac{J_s}{2}, \quad \dots (18)$$

There is no reduction in J_A .

3.6.3 Residual GSI Value and Strength Parameters

The residual GSI_r is a function of V_b^r and V_b^r which can be estimated using Equation 5.

As for the intact rock properties, fracturing and shearing do not weaken the intact rocks (even if they are broken into smaller pieces) so that the mechanical parameters (q_c and m_r) should be unchanged. Therefore, the generalized non-linear criterion for the residual strength of jointed rock masses can be written as:

$$\sigma_1 = \sigma_3 + q_c [m_{br} \frac{\sigma_3}{q_c} + s_r]^{n_r} \quad \dots (19)$$

where m_{br} , s_r and n_r are the residual constants for the rock mass. These constants can be determined from a residual GSI_r.

$$m_{br} = m_r \cdot \exp \left[\frac{GSI_r - 100}{28} \right] \quad \dots (20)$$

$$s_r = \exp \left[\frac{\text{GSI}_r - 100}{9} \right] \quad \dots (21)$$

$$n_r = \frac{1}{2} + \frac{1}{6} \left(e^{-\text{GSI}_r/15} - e^{-20/3} \right) \quad \dots (22)$$

Because the rock masses are already in a damaged, residual state, $D = 0$ is used for the residual strength parameter calculation.

3.7 Classification of Squeezing Ground Condition

Squeezing ground conditions on the basis of tunnel strain (u_a/a) or the ratio between rock mass strength and *in-situ* stress (γH), has been classified as shown in Fig. 4. In very severe squeezing ground ($u_a/a > 5$ percent), the tunnel face may exhibit the plastic extrusion due to the failure of rock mass all around the tunnel and face has to be stabilized. For a rock mass strength (q_{cmass} in Equation. 10) of 1.5 MPa and *in-situ* stress of 13.5 MPa (γH), the ratio ($q_{\text{cmass}}/\gamma H = 0.11$); Fig. 4 shows that this corresponds to a tunnel strain of 10 percent approximately and one should anticipate very severe squeezing ground condition. GSI is not applicable to the flowing and swelling grounds.

3.8 Effect of Intermediate Principal Stress

The rock mass strength in tunnels increases because of the effect of intermediate principal stress. Therefore, a polyaxial failure criterion shall be used for considering the effect of intermediate principal stress. Engineering judgement is needed for selection of strength and deformation parameters. The following polyaxial criterion is recommended for peak deviator stress at failure, for $0 < \sigma_3 < \sigma_2 < q_c$,

$$\sigma_1 - \sigma_3 = q_{\text{cmass}} + \frac{A(\sigma_2 + \sigma_3)}{2} - \frac{A(\sigma_2^2 + \sigma_3^2)}{4q_c} \quad \dots (23)$$

where

q_{cmass} = uniaxial rock mass quality;

$= 2c_p \cos \phi_p / (1 - \sin \phi_p)$;

$A = 2 \sin \phi_p / (1 - \sin \phi_p)$;

q_c = uniaxial compressive strength of rock material; and

= UCS for axial stress perpendicular to the planes of weakness in the anisotropic rock cores.

NOTE — In case of rocks, if UCS is estimated to be less than 5 percent of the UCS of rock material or 1 MPa, Equation 12 should be used in place of polyaxial criterion.

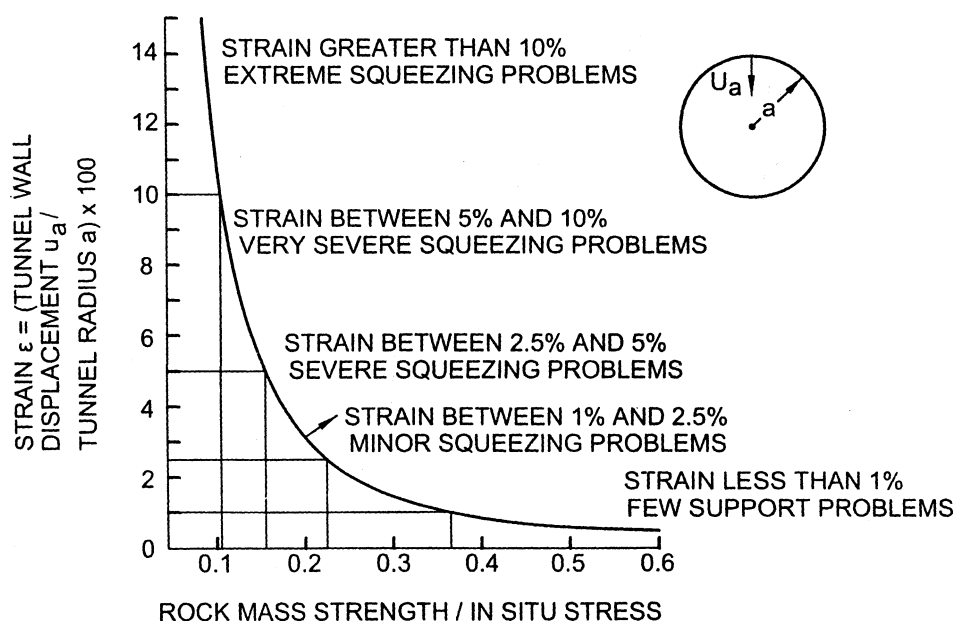


FIG. 4 TUNNELLING PROBLEMS ASSOCIATED WITH DIFFERENT LEVELS OF STRAIN

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Amendments Issued Since Publication

Amend No.	Date of Issue	Text Affected

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